SEISMIC ANALYSIS

FEDERAL BUILDING
517 Gold Avenue, SW
Albuquerque
New Mexico

BPLW Architects & Engineers, Inc.
Albuquerque, New Mexico
Architect's Project Number: 91062.004

TABLE OF CONTENTS

INTRODUCTION

INTRODUCTION:

The purpose of this report is to present and explain the seismic analysis of the Federal Building, 517 Gold Avenue SW, Albuquerque, New Mexico. This report was requested by and authorized by the General Services Administration, Design and Construction Branch, Fort Worth, Texas, through order number P-07-92-JU-0051 dated January 22, 1992.

SCOPE

SCOPE:

The seismic analysis of the Federal Building at 517 Gold Avenue consisted of a site investigation, a structural seismic analysis of the lateral force resisting system, and the preparation of several different options to upgrade the facility if the seismic analysis revealed any inadequacies. This report explains the procedure followed for each of these.

The initial site investigation of the building was performed by our office on January 31, 1992. This was a preliminary investigation to verify lateral load paths and structural dead loads indicated on the plans supplied by GSA. A second and more thorough investigation was performed on February 13, 1992. This investigation was to identify any areas of deterioration, note any nonstructural elements that could be of concern during an earthquake, examine the building for signs of foundation settlement, and confirm all structural dead loads.

The seismic analysis of the structure was performed according to the 1991 Uniform Building Code and the GSA Seismic Design Guidelines. The analysis consisted of identifying the lateral force resisting elements of the structure, calculating the forces in those elements, and comparing the calculated forces to the capacities of the elements.

Since the seismic analysis did reveal several inadequacies in the lateral force resisting system of this building, several different methods of upgrading the building were analyzed. The most economical options, and the options that require the least tenant disturbance, are presented in this report.

SITE INVESTIGATION

SITE INVESTIGATION:

The initial site investigation revealed that the building was built in general conformance to the plans provided by GSA. The building is an eight-story concrete structure with a mechanical room and an elevator equipment room on the roof (see photos 1-11, Appendix A). The plans revealed that the floors and roof of the building were two-way concrete slab systems. No evidence to the contrary was discovered during the initial site investigation although the floors were generally covered with carpet and the ceilings were inaccessible, making visual confirmation virtually impossible without some demolition.

The machine room and elevator equipment room on the roof were of the same construction as the rest of the building. The elevator equipment in the elevator equipment room (photos 12-14, Appendix A) were 1957 vintage and appeared to be quite heavy.

Two cooling towers (photos 15-18, Appendix A) are situated on the roof. These are supported by concrete beams. The concrete beams for one of the cooling towers was deflecting a great deal (see photos 19 & 20 , Appendix A). These are two-span beams supported by three concrete columns. The darker areas of the beams in photographs 17 & 18 reveal the locations of cracked portions of the beams that at the time of the photographs were penetrated with water.

The primary lateral force resisting elements, the concrete shear walls of the elevator shaft and the stair shafts, were observed from inside the stairways and from the elevator pit.

One of the primary issues prompting the second site visit was the question of whether or not the reinforcing steel in the concrete shear walls was continuous through the floor slabs. A closer look at the stair shafts revealed cold joints where the pour for the shear walls met the pours for the floor slabs (photo 21, Appendix A). However, there was no possible means of determining if the reinforcing was continuous through the cold joints.

A second reason for a return to the site was the apparent lack of any structural attachment of the exterior walls to the concrete frame. This was implied by the plans. It was not possible to observe the actual junction of the exterior structural clay tile and brick veneer wall to the concrete spandrel beam due to existence of a plaster suspended ceiling (photo 22, Appendix A). However, the screen wall on the roof (photo 23, Appendix A) was of the same construction as the exterior walls and this could be observed quite readily. As is clearly seen in photographs 24 & 25, the only "attachment" of the masonry to the concrete beam was a mortar joint. In fact, in photograph 25, it is possible to see where this mortar joint has come loose and the upper course of masonry has dislodged.

Model numbers and serial numbers of the cooling towers and the elevator equipment were obtained during the second site visit.

No real "deterioration" of the structural system was observed. However, a tour through the various floors of the building revealed that the floor slabs are deflecting noticeably (see photograph 26). In one particular area on the eighth floor, the floor deflects enough to cause a 1/2" gap at the bottom of the door frame on one side of the door while there is no gap on the other side of the door (see photo 27).

Evidence of differential foundation settlement was searched for in the basement and around the outside of the building. No such evidence was encountered. There were no major cracks in the basement floors or walls. No cracks were evident on the exterior of the building. All of the doors seem to function freely and none of the windows had stress-related cracks.

A third site visit was performed on March 24, 1992 to obtain more photographs as per GSA's comments on the pre-final report submittal.

SEISMIC ANALYSIS

SEISMIC ANALYSIS:

The seismic analysis, as mentioned earlier, was performed in accordance with the 1991 UBC and GSA's Seismic Design Guidelines. The static force methods as prescribed by the UBC was used for developing the seismic forces. The seismic analysis basically consisted of a code analysis with regard to the existing structure, generation of seismic dead loads, calculating the base shear of the building and distributing this force to the various floors, determining the amount of force in the lateral force resisting elements and comparing this force with the capacity of those elements.

CODE ANALYSIS - STRUCTURAL

CODE ANALYSIS - STRUCTURAL:

The seismic analysis criteria used follows the UBC procedure for determining the type of analysis and the amount of lateral force a structure must withstand. These are listed in Appendix B. The Federal Building is situated in seismic zone 2B, falls into occupancy category IV, and has no irregularities except for the machine room and elevator equipment room on the roof. The soil profile was assumed to be type S2. This determination was based primarily on the soil boring logs provided in the original plans.

The lateral force resisting elements of the structure were determined to be the concrete shear walls of the elevator and stair shafts (see shear wall plan, Appendix C). This was based on the assumption that the reinforcement in these walls is continuous from top to bottom. The structural system factor, Rw, was determined to be 8.

The concrete two-way slab and column system was investigated to determine if it could provide any lateral resistance according to code. The reinforcing details of the two-way slabs were in direct conflict with UBC chapter 26 sections 2625.k.6.D, 2625.k.6.E, and 2625.k.6.F. This makes the system an ordinary moment resistant frame which is specifically prohibited in seismic zone 2. GSA's Seismic Design Guidelines specifies that any existing structural system that does not conform to new construction requirements shall be given an appropriate Rw factor and designed accordingly. This was not prudent in this case for several reasons. The reinforcing details mentioned cause the slab systems in this building to have no ductility. If the slabs were used to resist lateral forces in an earthquake and did fail, the failure would be instantaneous and extremely catastrophic. In fact, GSA's own Seismic Design Guidelines lists this type of system as the second-most hazardous type of construction.

The exterior masonry walls were also investigated for the possibility of resisting lateral loads. These are nonload-bearing structural clay tile walls with a brick veneer. The UBC prohibits the use of nonload-bearing masonry units for lateral load resistance in section 2407.h.2.A. As mentioned earlier, an appropriate Rw factor could be applied to this wall and the corresponding forces in the wall calculated. Again this was not prudent in this case. There is no positive attachment of the clay tiles or brick veneer to the concrete frame. In all likelihood, the exterior walls would break loose and fall off the building during an earthquake. Even if there was a positive connection, relying on nonload-bearing, unreinforced masonry that has many discontinuities (windows) is not good practice. The Seismic Design Guidelines list unreinforced masonry as the most hazardous type of construction. Therefore, the only lateral force resisting elements of the structure used in this analysis were the concrete shear walls.

The first floor of the building has a granite panel veneer. This creates the appearance of a soft story. However, since the lateral force-resisting elements were determined to be only the concrete shear walls which are continuous to the mat foundation, the change in veneer had no bearing on the analysis. The change in veneer does not create a soft story.

LOAD GENERATION

LOAD GENERATION:

The UBC requires that the weight, W, be based on the structural dead load of the building and the weights of any fixed equipment. The structural dead loads generated from the building materials are listed in Appendix B. The weights of the various equipment, namely the cooling towers and elevator equipment were investigated by obtaining the model and serial numbers and contacting the manufacturers for the respective weights.

The manufacturer of one of the cooling towers, Baltimore Air Coil, provided the weight (64,000 lbs.) of the cooling tower.

The other cooling tower was manufactured by the Marley Co. Marley was contacted, but they could not provide the weight of that particular unit. The weight of the Marley cooling tower was estimated to be 75,000 lbs.

The Otis elevator equipment was originally estimated to be approximately 90,000 lbs. Subsequently, the manufacturer provided a weight of approximately 75,000 lbs. Since this would have caused a difference of less than .03% in the results, the original weight was used.

DETERMINATION OF BASE SHEAR

DETERMINATION OF BASE SHEAR:

As mentioned earlier, the only irregularity in the structure of the Federal Building was the machine room and elevator equipment room on the roof. The code allows for these to be analyzed separately and the results of this analysis to be superimposed with the analysis of the rest of the structure. The machine room and elevator equipment room were analyzed separately and the results from this analysis were superimposed with the analysis for the rest of the structure.

The base shear was determined by UBC equation 34-1. The shear was calculated both on the computer and by hand. The base shear was then distributed to the various floors by applying UBC equation 34-8.

Torsional effects of the rigid diaphragms due to the eccentricity of the center of mass and the center of rigidity was taken into account, including the 5% displacement of the center of mass prescribed by the UBC.

The amount of force resisted by each of the shear walls was calculated using the relative rigidity of each wall. This was done for each individual floor and then the forces in the respective walls from each floor were summed to determine the total force in each wall at the base of the building.

NON-STRUCTURAL ELEMENTS

NON-STRUCTURAL ELEMENTS:

Several non-structural elements caused some concern with regards to their behavior during an earthquake. These included the masonry screen wall on the roof, the support beams for the Baltimore Air Coil cooling tower mentioned earlier, and the original suspended plaster ceiling. There are many other non-structural elements, of course, but these were the only ones that appeared to pose a risk to life safety.

There is a possibility that the screen wall on the roof (photos 23 & 24) will collapse during an earthquake. It does not appear that there is any reinforcement in the wall and the wall has numerous cracks. This could cause a threat to life safety only if the portions of the wall that break off fall off the roof of the building to the street below, or if there would happen to be a person on the roof standing next to the wall when an earthquake occurred. Since the wall is located at least 25 feet from the edge of the roof and access to the roof is very limited, the possibility of loss of life from the collapse of the wall is very slight. Therefore, upgrading the wall to resist earthquake forces does not appear to be necessary.

The support beams for the Baltimore Air Coil cooling tower appear to have already failed structurally (see photos 17 - 20). Therefore, they should be strengthened so that they can withstand both gravity and earthquake loads. Detail #5 in Appendix D shows an inexpensive method of accomplishing this.

The original plaster ceiling (photos 22 & 28) was suspended from the structure with hanger wires supporting a steel suspension system. In general, this type of ceiling system would have to be braced with additional hanger wires to resist seismic forces. This is required by section 4704 of the UBC. However, footnote #7 of Table No. 23-P of the 1991 UBC states that such a system need not be analyzed to withstand seismic forces if the ceiling extends from wall to wall and the walls are not over 50 feet apart. This is exactly the case with the ceiling in the Federal Building and thus no upgrades were required.

CONCLUSIONS

CONCLUSIONS:

The analysis showed that the majority of the shear walls in the east-west direction would be severely overstressed in the event of an earthquake. The overstressed walls are shown on the last page of Appendix C, along with the amount of stress in each respective wall. The forces in these walls range from 120% of capacity to 260% of capacity. Even if the loads were reduced to 80% as recommended by GSA's Seismic Design Guidelines, the walls would still be overstressed as much as 240%.

Another area of concern was the connection of the exterior walls to the building frame. The UBC requires that the connection for such elements allows for a certain amount of movement. The existing "connection" does not do this. Without this freedom of movement, the exterior walls could break loose from the concrete frame and fall off the building in an earthquake.

The shear walls in the north-south direction are not overstressed.

RECOMMENDATIONS

RECOMMENDATIONS:

Given the calculated overstressed condition of the shear walls in the east-west direction, the structure should be upgraded with a supplementary lateral load resisting structural system for that direction. There are many different methods of accomplishing this. The next section of this report lists several different options that can successfully strengthen the building with little to no disturbance of the tenants.

Also required to reduce the hazard of this building to life safety is a connection of the exterior walls to the structural concrete frame. This can be accomplished with little disturbance of the tenants and very little change in the overall aesthetics of the exterior. This connection can be simply a continuous steel plate anchored to the spandrel beam at the tops and bottoms of the masonry walls and some angles connected to the bottom of the spandrel beam above the ceiling on the interior.

It does not appear that any other structural upgrades will be required to bring the building up to new life safety standards. However, we recommend that a gravity load analysis beyond the scope of this report be performed to determine the cause of the floor deflections mentioned earlier. This analysis should include the entire building and address all of the structural members supporting gravity loads.

CODE ANALYSIS - ARCHITECTURAL

CODE ANALYSIS-ARCHITECTURAL:

The following information was derived from drawings provided to us by GSA and the Uniform Building Code, 1991 edition.

The drawings provided the following information:

1. 8 stories with full basement
2. Gross area is approximately 270,000 SF
3. Location from property line is 3/4"

Based on the above information, the information below was extracted from the UBC.

1. Type of occupancy is B-2, office space.
2. Type of construction is Type I fire resistive
3. Exterior non-bearing walls to have a 4 hour fire resistive construction rating.
4. Structural frame to have a 3 hour fire resistive construction rating.
5. Exterior doors and windows: The building is located more than 20 feet from centerline of the public right-of-way, therefore openings are not required to be rated.

Per section 104 the existing building need not comply with the code. However, any new addition or alteration will be required to conform to the code. The new lateral resisting members will have to have a 3 hour fire resistive rating because section 1702 categorizes them as primary structure which is essential to the stability of the building. The alterations listed under the options sections will meet this section of the code.

OPTIONS

OPTIONS:

The options given below have been chosen from many different ideas. The criteria for selecting these options include:

1. Structural requirements
2. Code requirements
3. Cost effectiveness
4. Impact/disturbance of tenants
5. Aesthetic quality.

Each of the options has been analyzed structurally and the respective analyses are given in Appendix E. Itemized cost estimates for all of the options are given in Appendix F. These are very preliminary analyses performed primarily to provide a basis for the cost estimates for the different options. Sketches of the different options are provided in Appendix G to show the aesthetic effects of each on the exterior of the building.

There are several different drawbacks to the site conditions of this building that will have to be addressed when an upgrade is performed. The first one of these is the fact that the exterior face of the building is located only 3/4" away from the property line. This means that a zoning variance or an easement must be obtained to extend the structure beyond the property line.

A second drawback to any upgrade that requires excavation at the exterior of the building is the possible existence of underground utilities that may conflict with the new structural upgrade.

The options presented can basically be divided into three different categories according to the type of structural system employed by each. The first category uses an exterior concrete shear wall to resist lateral forces. The second category resists those forces through the use of an exterior steel moment resisting frame. The third category employs a unique concept of a panelized tube steel braced frame structure.

As requested by GSA's comments on the pre-final report submittal, Option 1A was investigated more comprehensively than the others. The structural analysis, conceptual details and cost estimate for Option 1A are given in Appendix D. Option 1A places the south shear wall over the property line. Since the existing foundation already encroached the property line, it was possible that an easement already existed. This investigation revealed that there is no existing easement, so securing an easement would be required before any upgrade could be performed. The cost of securing the easement has not been included in the cost estimates.

The locations of the existing underground utilities were also investigated after the pre-final report submittal. This was a preliminary investigation and should not be construed as a comprehensive investigation. Drawings showing the apparent location of underground utilities are given in Appendix H.

OPTION 1A:

The first option considered employs the use of a concrete shear wall located at approximately the center of the north and south walls (see sketch, Appendix G). This shear wall will be supported by a concrete footing poured at the same level and attached to the existing mat foundation as illustrated in detail #2, Appendix D. The size of this new footing as well as the footing reinforcement are only conceptual. A more comprehensive foundation analysis, including a new geotechnical investigation, may cause the footing design to change. Such design work is beyond the scope of this report. Each of the floors will be connected to the new shear wall to transfer the lateral forces into it (see detail #3, Appendix D).

The shear walls for this option were designed such that 100% of the code prescribed seismic forces would be resisted by both proposed and existing shear walls. The existing shear walls would be stressed to 74% of capacity and the proposed walls would be stressed to 90.5% of capacity. The proposed walls would be 50 feet long and 10" thick with two mats of #4 rebar at 9" on center each way.

The option of designing structural upgrades to resist 80% of prescribed seismic forces as provided for in GSA's Seismic Design Guidelines was investigated. This would reduce the amount of reinforcement in the proposed walls from #4's at 9" on center to #4's at 14" on center. As this reduction in reinforcement would be the only major cost saving aspect, the cost reduction would be only $10,893, or 0.75%. Changing the design to maximize the stress in the existing shear walls and to minimize the size of the proposed walls, which would result in the most economical design, was also investigated. Both of these would require that the length of the wall be reduced. Reducing the thickness of the wall would necessitate using only one mat of reinforcement and therefore would not be feasible. Reducing the length of the wall, however, would have a major detrimental effect on the aesthetics of the building, so these concepts were not pursued further.

Relatively little demolition would be performed for this option. The exterior wall, both structural clay tile and brick veneer, would be removed in the location of the new shear walls (see details). There are heating convectors located below the windows #1 & #1A, Appendix D that would have to be removed and replaced. After the new shear walls are in place, new interior metal stud and gypsum board walls would be installed. In areas not adjacent to the new shear walls, the existing exterior walls would be left in place and connected to the building frame as per detail #4, Appendix D. The existing spandrel beam covers (photo #30, Appendix A) appear to be made of concrete panels. There is a possibility that the concrete contains asbestos. These will be removed and disposed of properly. On the north side of the building, there is an existing handicapped ramp that will have to be moved approximately one foot away from the building to allow for the new shear wall (see photo #32, Appendix A).

Methods of minimizing the impact on the tenants would have to be investigated during the design of this option, but it seems entirely feasible to construct a temporary partition approximately 4 feet away from the exterior wall in the areas that are adjacent to the new shear wall. This would only affect several offices on each floor. As long as some interior access to the 4 foot wide construction zone was available, all construction activity could be performed within it. Of course there will be some impact on the tenants due to the general construction activity. This would probably include temporary shelters over the sidewalks to the doors of the building and a reduction of parking space on the north side of the building. At least one lane of Gold Avenue would have to be closed. In areas not adjacent to the new shear walls, the tenants would be disturbed by the installation of the connection angles for the exterior walls, but this could possibly be done after working hours. The only other disturbance for these areas would be the noise created by drilling and anchoring the plate shown in detail #4.

As per the GSA's comments on the pre-final report submittal, an investigation of the property easement situation mentioned earlier was also completed. This investigation revealed that there is no existing easement, so the procedure used to obtain one was researched. Basically, there are two ways to handle the situation: (1) vacation of public right of way and (2) an invocable permit. Vacation of public right of way is preferred by the City of Albuquerque. This would require an application be submitted to the Development review board. A surveyor will have to replatt the property and GSA will have to purchase a small section of land approximately 1 foot x 50 feet at the shear wall location. The land will be valued at current market value. Documentation of this investigation, as well as the procedure for vacation of public right of way are provided in Appendix J.

A subterranean encroachment agreement between GSA and the local government will have to be signed because the existing foundation system encroaches on the property line. The subterranean agreement will not require that GSA purchase any land.

The architectural impact of this option consists of a massive vertical element on the longitudinal sides of the building to break up the existing horizontal emphasis. Closer investigation of this option after the pre-final submittal revealed that utilizing a form liner to create a textured finish on the concrete shear wall would improve the general aesthetic appearance of the building, so this was included in the cost estimate.

OPTION 1B:

Option 1B utilizes the same structural concept as Option 1A. The difference between these options is the method of addressing the problem of the exterior wall. This option provides for the removal of the existing clay tile and brick wall and replacing it with a new curtain wall. This turned out to be the most costly option, but it will result in a very aesthetically pleasing, modern building. This option was presented to show the extreme spectrum of work required. This option addresses the aesthetic quality more than the cost effectiveness or impact to tenants.

Total estimated cost: $5,356,326

OPTION 2A:

This option falls into the second category described above, using an exterior steel frame to resist seismic forces. The stairstep configuration of the steel frame was designed to provide an aesthetically pleasing exterior appearance. In addition to the exterior frame, the exterior walls will be connected in the same manner as in Option 1A. The exterior frame will be fireproofed and covered with new architectural precast concrete fiber-reinforced panels.

Total estimated cost: $2,051,836

OPTION 2B:

This option also utilizes an exterior steel frame. However, instead of using only a partial frame as in Option 2A, this option provides for a steel frame covering the entire north and south faces of the building. The steel frame would then be fireproofed and covered with the same architectural panels as Option 2A. Again, the aesthetic effect of this option would be a new, modern facade for the building.

Total estimated cost: $3,367,995

OPTION 3A:

The first option of the third category uses prefabricated tube steel panels to provide lateral resistance. The brick veneer would be removed and replaced by these panels. The panels would be fireproofed and then the architectural concrete panels would be placed over the tube steel panels. The stairstep configuration of the panels would be similar to the partial steel frame of Option 2A. The aesthetic effect would be similar.

Total estimated cost: $2,021,197

OPTION 3B:

This option is similar to Option 3A in that it utilizes tube steel panels to provide resistance to seismic forces. It is also similar to Option 2B in that the tube steel panels would cover the north and south faces of the building in their entirety. The architectural effects would be similar to Option 2B.

Total estimated cost: $3,436,907